HIGHWAY RESEARCH REPORT

CORROSION AUTOPSY OF A STRUCTURALLY UNSOUND BRIDGE DECK

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16. ABSTRACT

An investigation was performed on a 12-year old deicing salt-contaminated reinforced concrete bridge deck which had to be replaced because of its deteriorated condition. In this investigation, the electrical half-cell potential measurements and chloride analysis were found to be related to the massive corrosion of the steel. The effect of chlorides present seems to be related to some threshold amount that changes the steel from a passive to active state. Beyond this point the amount of salt present has little or no effect except as it might influence the area of corrosion involved.

The chaining or sounding of the deck to locate delaminated concrete performed that function very well, but did not necessarily locate the corroded steel. From the observation of the type of cracking, it appeared that the final mode of distress was concrete fatigue. An investigation of actual concrete cover disclosed that there was reinforcing steel corrosion at depths greater than 3 inches.

It was determined that estimating the pit depth of steel by visually estimating the thickness of rust is not a very useful inspection technique. In this highly salt contaminated bridge deck, no relationship was found between variations in the chloride content of the concrete and the relative severity of the corrosion of the steel.

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November 1972

M&R No. 635116-8
Federal No. D-3-11

Mr. R. J. Datel
State Highway Engineer

Dear Sir:

Submitted herewith is a research report titled:

CORROSION AUTOPSY OF A STRUCTURALLY
UNSOUND BRIDGE DECK

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ABSTRACT: An investigation was performed on a 12-year old deicing salt contaminated reinforced concrete bridge deck which had to be replaced because of its deteriorated condition. In this investigation, the electrical halfcell potential measurements and effect of chlorides present seems to be related to some threshold amount that changes the steel from a passive to active state. Beyond this point the amount of salt present has little or no effect except as it might influence the area of corrosion involved. The chaining or sounding of the deck to locate delaminated concrete performed the function very well, but did not necessarily locate the corroded steel. From the observation of the type of cracking, it appeared that the final mode of distress was concrete fatigue. An investigation of actual concrete cover disclosed that there was reinforcing steel corrosion at depths greater than 3 inches.

It was determined that estimating the pit depth of steel by visually estimating the thickness of rust is not a very useful inspection technique. In this highly salt contaminated bridge deck, no relationship was found between variations in the chloride content of the concrete and the relative severity of the corrosion of the steel.

KEY WORDS: Corrosion, halfcell potential, chloride-ion, delamination, chain, fatigue, rust, deck replacement, salt, concrete cover, undersurface fracture.

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CORROSION AUTOPSY OF A STRUCTURALLY UNSOUND BRIDGE DECK

INTRODUCTION

After approximately 12 years of service, a highway bridge deck needed replacement because of the corrosion of reinforcing steel. The corrosion was associated with other factors such as thickness and quality of cover, presence of salt and moisture as will be discussed. Concrete cracking had advanced to the degree that falsework was necessary to prevent structural failure. As a result, it was decided to make a comprehensive investigation of the condition. This consisted of making electrical potential measurements[1,2], "chaining"[3], or "sounding"[4] the concrete for delamination, measuring the chloride content of the concrete, determining the relationship between amount of rusty steel and metal loss, and measuring the water absorption and strength of the concrete.

Although there are many reports in the literature[5] concerning bridge deck deterioration, there are no known reports that describe a comprehensive corrosion evaluation of a bridge that became structurally unsound and required deck replacement.

SUMMARY AND CONCLUSIONS

Electrical Potential Measurements

The electrical potential measurements were effective in indicating that the corrosion of the steel was of far greater extent than that which would be indicated by the measured area of concrete delamination. This type of measurement should always be included in any investigation of the corrosion of steel in a bridge deck.

The electrical potential measurements are not considered a reliable indicator of the rate or amount of corrosion of the steel. However, in general terms, the more extensive the area of active potentials, the more probable the greater amount of corrosion. This is because it takes time for a large area of corrosion to develop. Therefore, with a longer period of time for corrosion to be active, it is reasonable to assume that the corrosion loss would be greater. The halfcell potentials can be plotted on statistical distribution paper and used to "log" the condition of the bridge according to the percentage of active potentials. For example, for the bridge deck replaced, 94% of the potentials were measured to be active, while for an adjacent bridge in which about 2% of the deck area was repaired, only 30% of the values were found to be active.

Concrete Delamination

The use of the chain[3] as a sounding device to locate under surface fractures or concrete delaminations proved to be an effective and workable device. However, care must be exercised so as not to misinterpret the hollow sound as always representing corrosion caused concrete delaminations. In this investigation, one location that was thought to be a corrosion-caused concrete delamination actually turned out to be a location of disbonded epoxy membrane overlay. Delamination might also be the result of freeze-thaw action, or separation of a grout layer from the underlying concrete due to poor bond. In this bridge, just prior to replacement, approximately 20% of the total concrete deck surface was found to be delaminated.

Probable Cause of Deck Unsoundness

Although the cause of the structurally unsound deck is considered to have been accelerated by a somewhat porous concrete, it appears that the principal factors leading to the unsoundness are as follows:

1. Deicing salts were absorbed by the concrete which resulted in the corrosion of the steel.

2. The concrete was highly absorptive which apparently

resulted in some degradation of concrete strength, perhaps by a combined effect of salt crystallization within the concrete and freeze-thaw action. High water-cement ratios can greatly increase absorption and decrease strength of otherwise good concrete.

3. Corrosion of the steel caused concrete delamination or undersurface fractures and thus reduced the structural section of the deck to a point below that of the top mat of reinforcing steel.

4. As evidenced by the "alligator" type of cracking on the underside or soffit of the deck, the final stage of deterioration appeared to be fatigue failure of the slab which is a result of "live" loading.

Rust Rating and Pit Depth

An attempt was made to determine how well the actual corrosion pit depth would correlate with a visual observation of the quantity of rust on the steel. On a scale of 0, no rust, to 5, (see Table 3) visual observation of rust scale and metal loss of the deformations on the reinforcing steel, only a poor correlation could be made which was judged to be meaningless. Therefore, metal loss, if required, should be measured, not estimated by rust thickness.

Chloride Content

The chloride content of the concrete in the range encountered was not a positive indicator of deterioration as evidenced by the comparison of the similar amount of deterioration in the two spans that had different salt contents. The significance in a chloride analysis seems to lie within the spectrum of simply determining that its concentration is sufficiently great so as to cause steel to change from a passive to an active state, and thus be susceptible to corrosion. Beyond this point of chloride concentration, the control on the incidence of rust or corrosion rate will depend mostly upon the moisture content of the concrete[1]. Even if there are massive quantities of salt in the concrete, the steel will not actively corrode unless sufficient moisture is present. However, it is assumed that the extent of repairs would depend on the overall level of the chloride content. For example, in this structure, the chloride content was as great as 21 pounds of chloride-ion per cubic yard of concrete. Therefore, the use of a membrane to inhibit further intrusion of the salt into an already highly contaminated concrete would be useless.

Concrete Cover

The specified concrete cover over the steel for this bridge was 1-1/2-inch. By actual measurement, it was found that the average concrete cover was about 2.0 inches with a range of 1.5-inch to 3.0-inch. Corroding deck reinforcing steel was found not only in the top mat of steel where the concrete cover was as great as 3 inches, but also in the lower mat of steel where the concrete cover was greater than 6 inches. This could occur as the result of salt water reaching the steel by flowing downward through cracks.

Concrete Compressive Strength and Absorption

As indicated in Table 1 the compressive strength of the concrete was somewhat less than the strength level normally expected for 12-year old concrete and was variable. The relatively low concrete strength values coupled with the relatively high volumetric absorption of the concrete indicates that the deterioration of the structure was accelerated by a concrete quality initially inadequate for a freeze-thaw and salting exposure. This mechanism for the deterioration seems to be confirmed by the fact that Span 3 with its lower strength concrete showed distress, but falsework prevented actual failure while Span 1 with an equal amount of concrete delamination did not. However, it is not necessarily true that even with a better concrete quality, the service life could be greatly extended.

General Conclusions

No one investigative technique was found to be adequate to answer all questions concerning the cause or extent of the deterioration. In specifically dealing with the corrosion phenomenon, the electrical halfcell potentials and the chloride analysis of the concrete appear to be the most important for determining the extent and level of the corrosion activity. The electrical potential measurements have been previously found to be useful for predicting the locations of new concrete spalls[1]. Conversely, for determining the area and location of advanced structural damage resulting from a concrete delamination, the "drummy" sound of the chain when dragged over delaminated concrete was found to be the easiest and most rapid technique for locating undersurface fractures or spalls.

Because corrosion was found in all previous repairs, the epoxy mortar repairs did not inhibit further corrosion of the steel.

Since corrosion was found when the concrete cover was as great as 3 inches, it is apparent that an even greater depth of cover would be required for the concrete quality found in this bridge for the protection of the steel from salt.

BRIDGE HISTORY

The Truckee River Bridge, No. 1713R, is a 3-span bridge of approximately 100 feet per span with a traveled way of 28 feet curb to curb. It carries two lanes of eastbound traffic. The deck slopes to the right. It is a welded steel composite girder structure with four girders per span and reinforced concrete pier and wing abutments. All of the reinforced 6-sack concrete was designed to contain 4% to 6% entrained air. The concrete aggregate was blended from a single source; the coarse aggregate (1-1/2-inch maximum size) had a water absorption of about 2.8% by weight, while the fine aggregate had a water absorption of about 4% by weight.

The average 28-day concrete strength for the eleven bridges being constructed at the same time was 3800 psi. There was only one test report for one concrete cylinder that could be positively identified as coming from the Truckee River Bridge. The 28-day compressive strength for the one concrete cylinder was 2940 psi.

The cement used was an ASTM Type II of low-alkali content meeting California Highway Specifications which, in various shipments, was either ground to ordinary range of fineness or was finely ground to provide high early strength. The records are not clear as to when either kind of cement was used, but it is assumed that the finer type was used during periods of low ambient temperatures to accelerate early strength gain.

The bridge is located in the Sierra Nevada Mountains at an elevation of approximately 5500 feet above mean sea level. The average rainfall is reported as about 24 inches. Most of the precipitation is in the form of snow as the average annual snowfall is about 170 inches. The temperature range in the area is from about 95°F to 41° below zero as measured in 1949. The frost penetration in the soil was anticipated at a 4-foot depth. As a consequence, the bridge and roadway is heavily salted during the winter season due to snowfall and frost. This bridge was completed in 1959, and was inspected at least annually by an engineer as part of a regular inspection program. Additional inspections were made as warranted. A review of the reports of inspections draws an interesting picture of the progressive deterioration of the deck of the structure.

In the report dated September 1960, it was observed that several large transverse cracks have opened up in all three spans as well as numerous other smaller cracks in the deck. In September 1961, it was observed that the transverse cracks in the three spans was "medium" in Span 3 and "light" in the other two spans as well as "medium" random pattern cracking in all spans.

In September of 1963 it was reported that after only four winters, there was considerable "pattern cracking" of the soffit or the bottom of the deck of Span 3 between the third and fourth girders. The following summer, the bridge was overlaid with an epoxy-sand seal. In August of 1967, the soffit cracking in Span 3 was described as severe. Leaking of water through cracks in the deck extended for over 20 feet, and cracking was spreading to the deck areas between the second and third girders. The engineer estimated that this area of the deck would require replacement in one to two years. It was also noted in this 1967 report that seven areas of the deck had spalling and that the soffit of Span 1 was starting to show cracking.

In May of 1968, the soffit cracking in Span 3 had increased to the point that its structural soundness was in question and the placement of timber supports was recommended. The placing of the timber supports as well as the filling of 35 new deck spalls with an epoxy mortar and placing a new epoxy-sand seal in the slow traffic lane was completed during the summer of 1968. The epoxy mortar was made with a ratio of 5 parts pea gravel to one part (1) epoxy by volume.

Additional deck spalling occurred and was repaired prior to the October 1969 report in which it was recommended that the eleven additional spalls be repaired and the bridge deck be scheduled for replacement.

FIELD WORK

Initially, a reference grid was laid out on the deck on a 4-foot square pattern. These points were spotted on the deck using spray paint. The deck was then chained to delineate the areas of unsound or delaminated concrete. A solid ringing sound is normally heard as the chain is dragged on sound concrete, but there will be a dead or drummy sound when delaminated areas are encountered. The delaminated areas were outlined on the deck using spray paint and then they were plotted on cross-section paper for correlation with other operations. The existing deck patches of epoxy mortar and asphalt concrete, and the areas of the deck being supported by timber falsework were also plotted on this sheet. Figures 1 and 2 show portions of Spans 1 and 3 with delaminated areas being shaded and patched areas crosshatched.

Electrical potential measurements using a saturated copper-copper sulfate halfcell were taken on the 4-foot grid pattern on the top of the deck with additional readings at any anomalies. These readings were reduced to contours and overlaid on the plot showing the delaminated areas. Figures 1 and 2 show these contours as a dashed line.

Twenty-two 4-inch cores were taken through the deck at various locations. The core locations were chosen to include all stages of deterioration and all ages of epoxy mortar patches. These locations were further chosen to include deck reinforcing. All cores were identified and their locations recorded. They were then analyzed for compressive strength, absorption, and chloride content.

The first phase of the deck removal consisted of removing the concrete from both sides of both exterior girders by striking the concrete with a pneumatic mounted hydro-hammer. This operation exposed the reinforcing steel from the curb lines to about two feet out into the traveled way. It was observed that a fairly large percentage of this steel showed from minor to extensive corrosion. However, the chaining had indicated very little concrete delamination along these areas even though the halfcell potential readings indicated active corrosion.

During the concrete removal, the actual amount of concrete cover over the steel was measured. Along the left (facing the direction of traffic movement) edge of the deck, it was found that the average concrete cover over the steel was 2.36-inch with a range of 2.0 to 3.0-inch. The concrete cover over the steel along the right edge (lower) of deck averaged 2.25-inch with a range of 1.5 to 2.75-inch. These measurements were made with a ruler.

Along the middle of the deck, a pachometer was used for measurement and the indicated average amount of concrete cover over the steel was 1.78-inch with a range of 1.63 to 2-inch.

The contractor was removing the deck in slabs approximately 7x12-feet and with his cooperation, three selected slabs from specific areas were set aside for recovery of the reinforcing steel. The slabs were from (1) an area of little delamination or cracking of the concrete; (2) an area that showed severe cracking on the soffit and epoxy patches of different ages on the top of the deck; (3) the timber-supported area which was structurally unsound. Upon removal, the reinforcing steel was identified as to its location. A detailed corrosion evaluation of the steel was made. The locations of these three test slabs are shown on Figures 1 and 2.

The abutments and piers were examined visually for evidence of corrosion related deterioration. Pier 3 showed the only visual evidences of corrosion in that there was corrosion-caused spalling of concrete on the top and bottom of the pier cap. The top of the pier cap sloped with the cross-slope of the deck and corrosion-caused deterioration was found only on the low end of the cap, where salt contaminated deck drainage water would flow. Electrical potentials were taken on the top of the cap. At one location the average of four readings was -0.24 volts and there was no visible evidence of corrosion. At another location, the average potential was -0.53 volts, and corrosion of the steel and concrete spalling was observed.

Samples of concrete were taken in the area of high potentials for chloride analysis. The results of this analysis showed the concrete at the top of the cap to contain the equivalent of 31 lbs./cu.yd. of chloride-ions and the bottom to contain 10 lbs./cu.yd.

CONCRETE QUALITY

Twenty-two 4-inch diameter cores were taken through the deck for laboratory analysis, ten cores from Span 1 and twelve from Span 3. Ten cores were in areas of deck spalling that had been repaired with epoxy mortar. All ten cores through patches and eight cores in unpatched areas were taken so as to include deck reinforcing. The reinforcing was found to be corroding at all epoxy patches. When making epoxy repairs, it is required that the steel be sandblasted, therefore, it is assumed that any corrosion products present must have been generated as a result of corrosion occurring after the time of repair.

Four cores, two each from Spans 1 and 3, contained neither epoxy patches nor reinforcing steel. They were tested for 28-day absorption using California Test Method 538-A. These four cores were then checked for compressive strength (Table 1) which appear to be lower than would be expected for 12 year old concrete. Table 1 shows the 28-day volumetric absorption of the concrete between 14.09 and 16.61%. In previous testing, the authors have generally observed that in similar concrete mixes, the volumetric absorption would be in the range of 13.5 to 15%. The greater absorption of the bridge deck concrete is probably influenced by the absorptive aggregates.

The 22 cores were then analyzed for chloride-ion intrusion. Each core was cut into 1-inch thick disks which were pulverized and analyzed using a "wet" analysis for control and X-ray diffraction (Table 2).

Table 2 shows that a significant quantity of chloride-ion has been absorbed by the concrete. Even though the chloride content was greater in Span 1, there was no difference in the area of delamination between this span and Span 3 which was structurally unsound.

In previous work[1], it was observed that as long as the salt content was sufficient to support corrosion, then the presence or absence of corrosion depended upon the moisture content of the concrete.

The threshold value of salt as measured for previous work was found to be about 1.5 lb. of chloride per cubic yard of concrete. Of course localized distribution can vary greatly.

It seems clear that once the minimum level of salt has been reached to cause corrosion, even greater concentrations play no further significant role in the corrosion of the steel. This is emphasized by comparing the chloride concentrations in the deck of this bridge, about 13 pounds, to a previously reported[1] bridge that was corroding and eventually removed, and only contained an average of about 1.5 pounds of chloride-ion per cubic yard at the level of the steel.

HALFCELL POTENTIALS, CONCRETE DELAMINATION, AND RUSTING STEEL

In previous work [6], it was shown that for halfcell potential values more negative than about -0.35 volts to the copper-copper sulfate halfcell ($Cu \cdot CuSO_4$), the steel in concrete appears to be active as a result of salt intrusion. The range of -0.30 to -0.35 volts seems to be a gray area while for values less negative than -0.30 volts ($Cu \cdot CuSO_4$), the steel is passive or chemically inhibited from corrosion.

To determine the relationship between the halfcell potentials, rusting steel and concrete spalls or delamination, the bridge was completely surveyed by electrical potential measurements, chained to find loose or delaminated concrete, and visually observed to locate any evidence of rusty steel.

Figures 1 and 2, show that the high potential measurements on nearly the whole deck surface indicated a massive area of active corrosion of the steel. Although not shown on Figures 1 and 2, visual observations of the steel during the concrete removal show significant areas of rusting steel even though the concrete in these areas showed no indication of spalling or delamination.

For the entire bridge deck about 94% of all of the halfcell potentials of the steel were in the active or corroding range. In contrast, approximately 20% of the deck area was found to be delaminated or to have undersurface concrete fractures at the top layer of the steel.

Figure 3, shows the electrical potentials and concrete delamination that was measured on the relatively good slab. All of the halfcell potentials are in the active range of -0.35 to -0.55 volts and denote the likelihood of significant corrosion of the steel. On Figures 3, 4, and 5 the locations of steel that had visible rust was plotted as solid lines. If the top mat of reinforcing had no rust, it was not shown.

Shown on Figure 3 is an undersurface delamination that apparently is in a location of unrusted steel. Because this area of concrete still had the epoxy-aggregate membrane on the surface, it is believed that the hollow sound that caused a recording of a concrete delamination might have been the result of disbonding between the epoxy membrane and the underlying concrete surface.

As shown by Figures 3, 4, and 5, the relationship between rusted steel and concrete delaminations can be poor. This is not to say that the corroding steel does not cause delaminations, but it does indicate that the amount of rust that forms on steel causing a spall can be highly variable. It is obvious that the sounding of concrete only relates to the condition of the concrete and not necessarily to the condition of the steel.

To relate the halfcell potential measurements to the observed condition of the bridge, Figure 6, shows the cumulative frequency distribution on three bridges. The distribution curve "Replace Deck" is for the bridge under investigation, while the distribution curve for another bridge is titled "Repair 2% Deck Area". For the bridge deck that required replacement, about 94% of all of the potential measurements made showed that the steel was active (more negative than -0.35 volts, Cu·$CuSO_4$) or corroding. For the deck that was repaired, only 30% of the measurements showed active halfcell potentials, and the new deck had no measured active potentials.

The data shown on Figure 6 may reflect a means for "logging" the condition of a bridge according to the percentage of active or passive potentials. However the distribution curve can have a "break", therefore one must exercise caution in mathematically calculating a mean or standard deviation for the halfcell potentials.

REINFORCING

The reinforcing steel removed from the three slabs recovered from the deck was evaluated as to degree of corrosion on a visual rating of 1 to 5 (Table 3). The slabs contained four layers of steel, two layers of top steel, with the upper layer transverse, and the second longitudinal, and two layers of bottom steel, the upper being longitudinal and the bottom transverse. Truss bars were considered top steel when in that plane, and bottom steel when in that plane.

Table 3, shows that the amount and severity of rusting increased with the severity of the original physical condition of the test slabs. Slab No. 1 was from the area in better physical condition, while Test Slab No. 3 was from the area supported by falsework.

In Test Slabs 2 and 3, there is a significant length of bottom reinforcing that was found to have significant rusting.

To determine the relation of a visual rating of rust to the actual amount of metal loss, random pieces representing each visual rust rating were sandblasted and maximum pit depths measured. It became obvious that estimating and logging degrees or amounts of rust can be misleading. Pit depths for 20 samples of each visual rating varied as shown in Table 4.

The amount of metal loss to produce a visual volume of rust is highly variable as indicated in Table 4. This variable in amount of rust produced further illustrates the lack of a significant relationship between delaminated concrete and rusted steel. In concrete of low absorption where small amounts of rust will cause high disruptive pressure, (no absorption of the rust products by the adjacent concrete) the relationship between rusty steel and concrete delamination should be of a high order.

Table 5 shows the average potential, $\overline{X}$, the percent of the potentials that were active, the area of concrete delamination, and the percentage of rusted steel have all increased in value commensurate with the increase in proportion to the distress observed in the slabs. The halfcell potentials were a better indicator of the area of rusted steel than was the area of concrete delamination, as is especially emphasized in the data from Slab No. 1 wherein 91% of the halfcell potentials measured were found to be in the active range while only 4% of the concrete was delaminated by rusting of the top steel which was coated with rust for 29% of its length.

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TABLE 1

28-Day Absorption/Compressive Strength

4-Inch Cores

Spans 1 & 3

Core No.	Compressive Strength, psi	28-Day Absorption Percent/Vol.	Location
2	3690	16.61	Span 1
3	4080	16.38	Span 1
11	3020	14.42	Span 3
20	2695	14.09	Span 3

TABLE 2

Average Chloride-ion Distribution as a

Function of Slab Depth*

Depth, In.	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8
Span 1	23.5	13.2	4.4	1.3	1.4	0.9	1.3	0.5
Span 3	18.3	12.4	2.9	0.9	0.6	0.7	0.3	---

*Calculated as pounds of chloride-ion per cubic yard of concrete. 0-1 is top one inch of deck surface.

TABLE 3

Length of Rusting in Three Deck Slabs*

		Rusting Rating				
Slab No. 1	No Rust	#1	#2	#3	#4	#5
(Better Condition)						
Top transverse bars	71	13	3	4	5	4
Top longitudinal bars	99	<1	1	--	--	--
Bottom longitudinal bars	100	--	--	--	--	--
Bottom transverse bars	99	<1	1	--	--	--
Slab No. 2						
Top transverse bars	41	12	3	4	3	37
Top longitudinal bars	81	8	2	1	1	7
Bottom longitudinal bars	94	1	<1	1	1	3
Bottom transverse bars	86	9	3	2	<1	<1
Slab No. 3						
(Unsound)						
Top transverse bars	19	7	2	6	4	62
Top longitudinal bars	48	5	3	5	8	32
Bottom longitudinal bars	79	1	1	2	3	14
Bottom transverse bars	50	18	5	9	11	7

*In percent of total length of the steel

For Rust Rating #5 represents heavy rusting

TABLE 4

Visual Rusting and Actual Pitting in Mils

Rating	Pit Depth Range	Avg.	Visual Condition
No. 1	0 to 46	15	Trace to light rust
No. 2	0 to 57	17	Medium to heavy rust
No. 3	3 to 58	18	Very heavy rust
No. 4	5 to 57	22	Light pitting and corrosion of deformations
No. 5	21 to 97	52	Heavy pitting and corrosion of deformations

TABLE 5

Average Potential Values Found in Test Slabs

	Slab No. 1	Slab No. 2	Slab No. 3
$\bar{X}$ Potential (Volts)	-0.44	-0.47	-0.53
% Active Potential*	91	98	100
Concrete Delamination, % of total area	4	34	58
% Rust - top bar - of total length	29	59	81
% Rust - All bars - of total length	19	28	55

* % of total measurements that exceed -0.35 volt, $Cu \cdot CuSO_4$

EQUI-POTENTIAL CONTOURS &
UNDERSURFACE FRACTURES

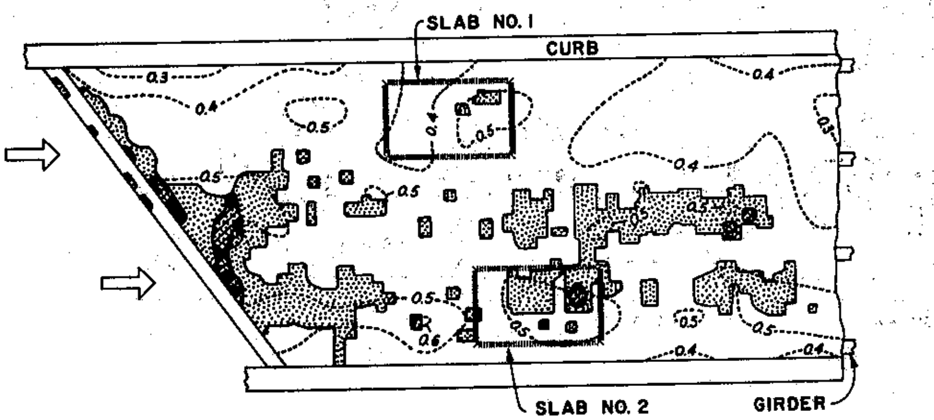

Figure 1. Part of Span 1

Shaded area ---------------- Delaminations

Cross-hatched area --------- Epoxy patches

Dashed lines --------------- Electrical potential
contours

EQUI-POTENTIAL CONTOURS &
UNDERSURFACE FRACTURES

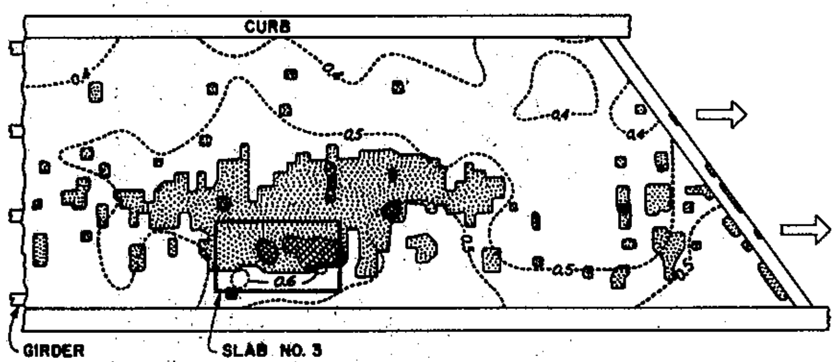

Figure 2. Part of Span 3

Shaded area ---------------- Delaminations

Cross-hatched area --------- Epoxy patches

Dashed lines --------------- Electrical potential contours

EQUI-POTENTIAL CONTOURS & UNDERSURFACE FRACTURES

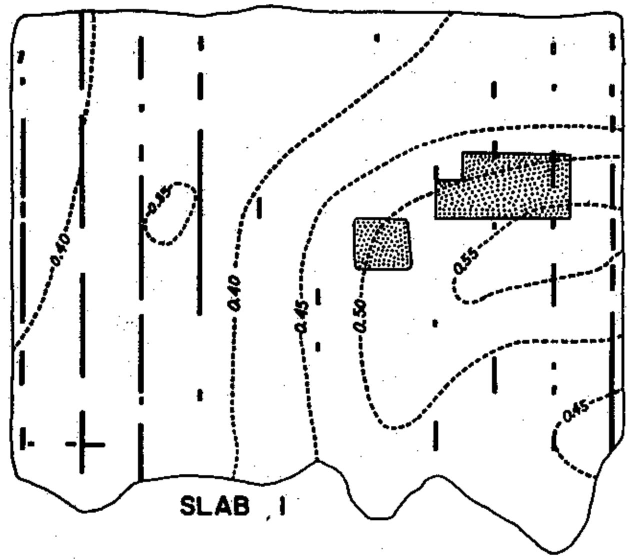

Figure 3. Slab showing little delamination.

Shaded area ------------- Delaminations

Dashed lines ------------ Electrical potential contours

Solid line ------------- Corroded top reinforcing steel

EQUI-POTENTIAL CONTOURS & UNDERSURFACE FRACTURES

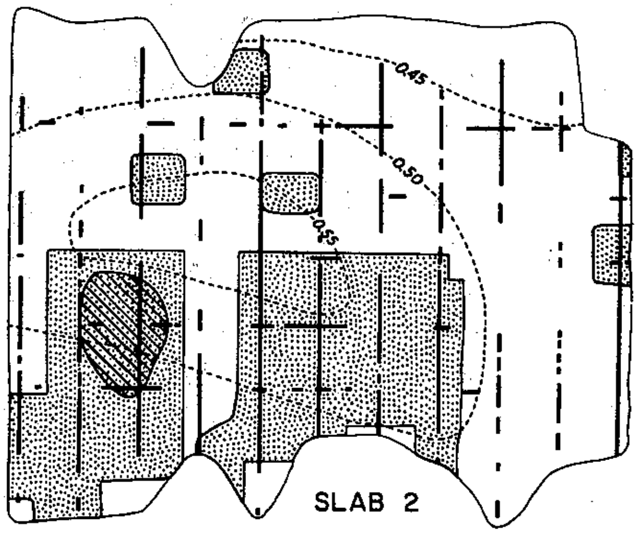

Figure 4. Slab showing severe cracking of soffit and epoxy patches.

Shaded area ------------------ Delaminations

Cross-hatched area ----------- Epoxy patch

Dashed line ------------------ Electrical potential contours

Solid line ------------------- Corroded top reinforcing steel

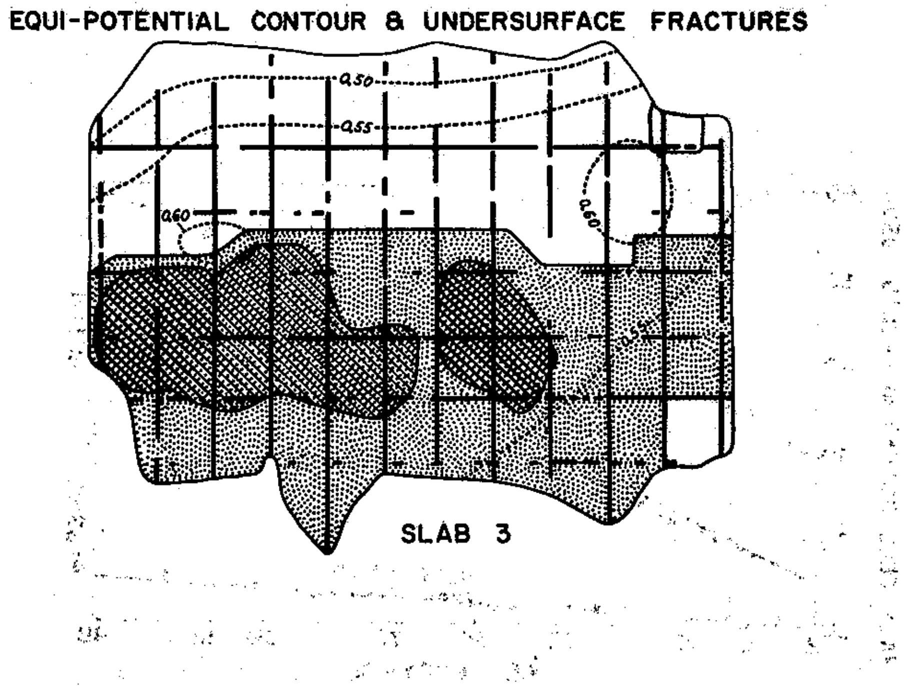

Figure 5. Slab from area of timber supports.

Shaded area --------------- Delaminations

Cross-hatched area -------- Epoxy patches

Dashed lines -------------- Electrical potential contours

Solid lines --------------- Corroded top reinforcing steel

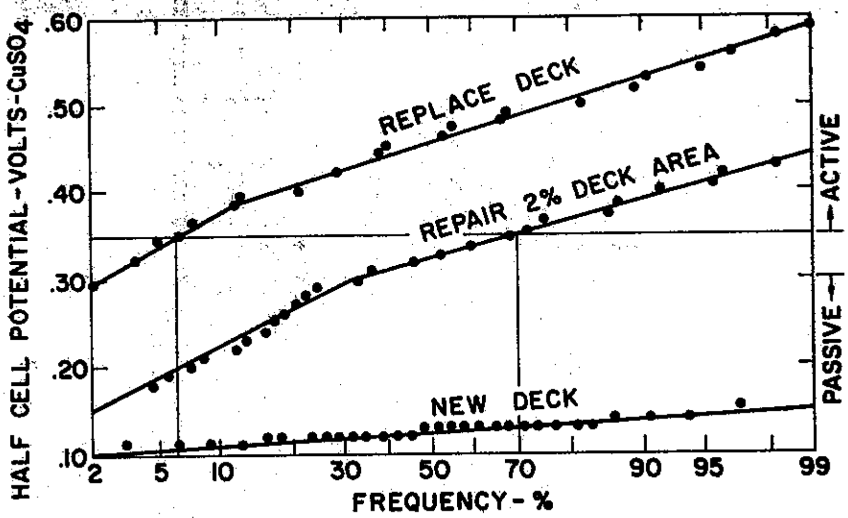

Figure 6. Distribution of halfcell potentials.